US Army Corps of Engineers

TECHNICAL REPORT HL-89-19

MODEL STUDY OF PRADO FLOOD-CONTROL DAM

Hydraulic Model Investigatior

by

John F. George

Hydraulics Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

September 1989

Final Report

HYDRAULICS LABORATORY

Prepared for US Army Engineer District, Los Angeles
Los Angeles, California 90053-2325

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

Form Approved
OMB No 0704-0188

1a. REPORT SECURITY CLASSIFICATION
Unclassified

1b. RESTRICTIVE MARKINGS

2a. SECURITY CLASSIFICATION AUTHORITY

2b. DECLASSIFICATION/DOWNGRADING SCHEDULE

3. DISTRIBUTION/AVAILABILITY OF REPORT
Approved for public release; distribution unlimited.

4. PERFORMING ORGANIZATION REPORT NUMBER(S)
Technical Report HL-89-19

5. MONITORING ORGANIZATION REPORT NUMBER(S)

6a. NAME OF PERFORMING ORGANIZATION
USAEWES
Hydraulics Laboratory

6b. OFFICE SYMBOL (If applicable)
CEWES-HS-L

7a. NAME OF MONITORING ORGANIZATION

6c. ADDRESS (City, State, and ZIP Code)
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

7b. ADDRESS (City, State, and ZIP Code)

8a. NAME OF FUNDING/SPONSORING ORGANIZATION
USAED, Los Angeles

8b. OFFICE SYMBOL (If applicable)

9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER

8c. ADDRESS (City, State, and ZIP Code)
PO Box 2711
Los Angeles, CA 90053-2325

10. SOURCE OF FUNDING NUMBERS

PROGRAM ELEMENT NO	PROJECT NO	TASK NO.	WORK UNIT ACCESSION NO

11. TITLE (Include Security Classification)
Model Study of Prado Flood-Control Dam; Hydraulic Model Investigation

12. PERSONAL AUTHOR(S)
George, John F.

13a. TYPE OF REPORT
Final report

13b. TIME COVERED
FROM ________ TO ______

14. DATE OF REPORT (Year, Month, Day)
September 1989

15. PAGE COUNT
42

16. SUPPLEMENTARY NOTATION
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

17. COSATI CODES

FIELD	GROUP	SUB-GROUP

18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number)
Hydraulic models
Prado Dam
Santa Anna River
Spillways

19. ABSTRACT (Continue on reverse if necessary and identify by block number)

Tests were conducted on a 1:80-scale model of the Prado Dam and spillway to determine the adequacy of proposed modifications to the existing structure to convey the revised design flow of 615,000 cfs. These modifications were to serve as an interim solution until decisions were made either to make major design modifications to the existing structure or construct a new structure upstream of Prado Dam that would control downstream flow releases.

Unsymmetrical approach conditions to the spillway resulted in poor flow conditions at the left and right abutments at the weir, which caused a reduction in the effective length of the weir. Test results indicated that installing long approach walls on either side improved flow conditions at the weir, but did not significantly increase the capacity

(Continued)

20. DISTRIBUTION/AVAILABILITY OF ABSTRACT
☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT ☐ DTIC USERS

21. ABSTRACT SECURITY CLASSIFICATION
Unclassified

22a. NAME OF RESPONSIBLE INDIVIDUAL

22b. TELEPHONE (Include Area Code)

22c. OFFICE SYMBOL

19. ABSTRACT (Continued).

of the spillway or improve flow conditions on the spillway. Therefore, dikes were installed on the left and right abutments to streamline approach conditions to the spillway, thereby improving flow conditions at the weir.

With design conditions the water surface exceeded proposed wall heights along the spillway. Additional wall height was obtained by paving a small berm and existing topography along each side of the spillway. A wall having a warped surface also had to be placed on top of the berm at the downstream end of the spillway chute to redirect any flow that overtopped the paved berm back onto the spillway.

Flow conditions downstream of the flip bucket were unsatisfactory, with large standing waves present in the exit channel and a large eddy present between the spillway exit channel and the earth dam. Maximum velocities of 11 fps were recorded along the toe of the dam with a discharge of 400,000 cfs. Due to the erodibility of the material in the exit channel, the potential is high for severe scour to occur in these areas with a major flood event. No noticeable backwater effects were observed from the constricted bridge crossing and highways just downstream of the spillway. The design discharge of 615,000 cfs simply submerges these structures.

PREFACE

The model investigation reported herein was authorized by the US Army Engineer Division, South Pacific (SPD), on 12 March 1981, at the request of the US Army Engineer District, Los Angeles (SPL). The study was conducted by personnel of the Hydraulics Laboratory (HL), US Army Engineer Waterways Experiment Station (WES), during the period January 1981 to June 1983.

All studies were conducted under the direction of Messrs. H. B. Simmons and F. L. Herrmann, Jr., former and present Chiefs, HL; and J. L. Grace, Jr., Chief of the Hydraulic Structures Division. The tests were conducted by Messrs. J. F. George, J. H. Riley, and C. L. Dent, all of the Locks and Conduits Branch, under the supervision of Mr. G. A. Pickering, Chief of the Locks and Conduits Branch. This report was prepared by Mr. George and edited by Mrs. Marsha Gay, Information Technology Laboratory, WES.

Messrs. S. B. Powell, Headquarters, US Army Corps of Engineers; Ted Albrecht, SPD, and Bob Koplin, SPL, visited WES during the study to discuss test results and to correlate these results with concurrent design work.

Acting Commander and Director of WES during preparation of this report was LTC Jack R. Stephens, EN. Technical Director was Dr. Robert W. Whalin.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	by	To Obtain
acres	4,046.873	square metres
acre-feet	1,233.489	cubic metres
cubic feet	0.02831685	cubic metres
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres
square feet	0.09290304	square metres
square miles	2.589988	square kilometres

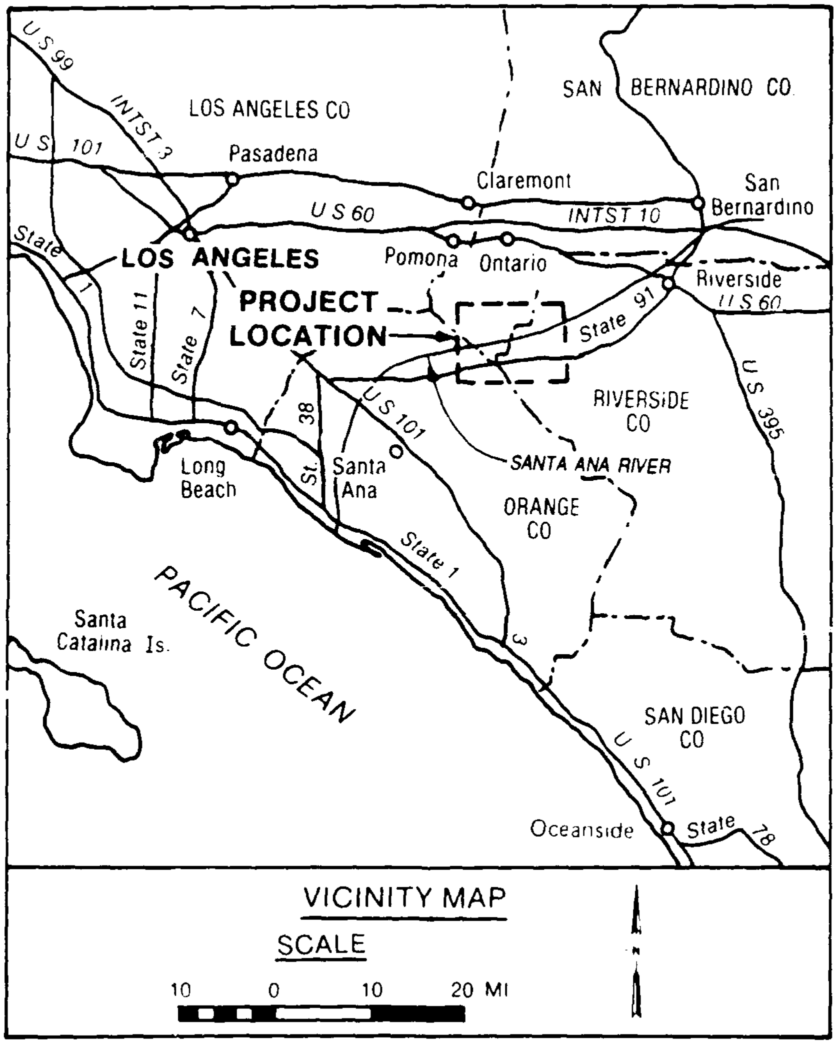


Figure 1. Vicinity map

MODEL STUDY OF PRADO FLOOD-CONTROL DAM

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. The Prado Dam and spillway is located on the Santa Ana River about 30 miles* from the river's mouth at the Pacific Ocean and in the upper extremity of Santa Ana Canyon (Figure 1). The tributary drainage system upstream of the dam, comprising an area of 2,255 square miles of mountains, foothills, and valley floor, is situated in San Bernardino, Riverside, and Los Angeles Counties. The elevation of the drainage area varies from 470** at the reservoir to 11,485 at San Gorgonio Peak in the San Bernardino Mountains.

2. The spillway (Plate 1) consists of a concrete ogee-type structure that is 13 ft high with a crest length of 1,000 ft at el 543. The spillway is 1,100 ft long and converges to a 660-ft width with a flip bucket at the downstream end.

3. The rolled earth-filled dam (Plate 2), having a crest length of 2,280 ft at el 566 and a maximum height of 105 ft above streambed, forms a reservoir with an area of 10,830 acres and a corresponding capacity of 423,000 acre-feet at the top of the dam.

4. The outlet structure (Plate 2), which is located within the earth-filled dam near the right abutment, consists of an intake tower with two gated conduits with gate sills at el 460, and two ungated circular conduit bypasses, 5.5 ft in diameter with the entrance invert at el 462. The combined maximum capacity of the conduits is 17,300 cfs with the pool stage at spillway crest.

Project Design Flood

5. The spillway was originally designed for a discharge capacity of

---

* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

** All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

178,000 cfs upon its completion in 1941. Due to increases in reservoir and spillway design rainfalls as a result of revised hydrologic analyses, and the increase in runoff to the reservoir due to unanticipated urbanization over much of the valley, the spillway design discharge was increased to 615,000 cfs.

Purpose of Model Study

6. Modifications proposed for the existing dam and spillway were to serve as an interim solution until decisions were made either to construct a new dam upstream of Prado Dam or make major modifications at Prado Dam in order to control releases from a major flood event. The initial plan of improvements consisted of raising the dam embankment from el 560 to el 581.4 and spillway walls of the existing structure to safely pass the Probable Maximum Flood. A model study was considered necessary to verify the adequacy of the existing spillway design along with proposed modifications and develop, if necessary, a design that would provide satisfactory flow conditions throughout the structure. Specifically, the model study was to

- a. Document flow conditions along the spillway.
- b. Determine flow conditions in the downstream vicinity of the spillway.
- c. Evaluate backwater conditions resulting from the constricted bridge crossings and highways just downstream of the spillway.

PART II: THE MODEL

Description

7. The 1:80-scale model (Figure 2, Plate 2) reproduced approximately 1,500 ft of approach channel to the spillway, the spillway (Photo 1) and dam, the outlet works stilling basin and intake tower (Photo 2), and approximately 3,400 ft of exit channel. The spillway was constructed of sheet metal, and the intake tower and outlet works stilling basin were constructed of transparent plastic. The approach and exit channels and dam were molded in sand and cement mortar to sheet metal templates.

Model Appurtenances

8. Water used in the operation of the model was supplied by a circulating system. Discharges were measured with venturi meters installed in the flow lines and were baffled before entering the model. Velocities were measured with pitot tubes that were mounted to permit measurement of flow from any direction and at any depth. Water-surface elevations were measured with point gages. Different designs and various flow conditions were recorded photographically.

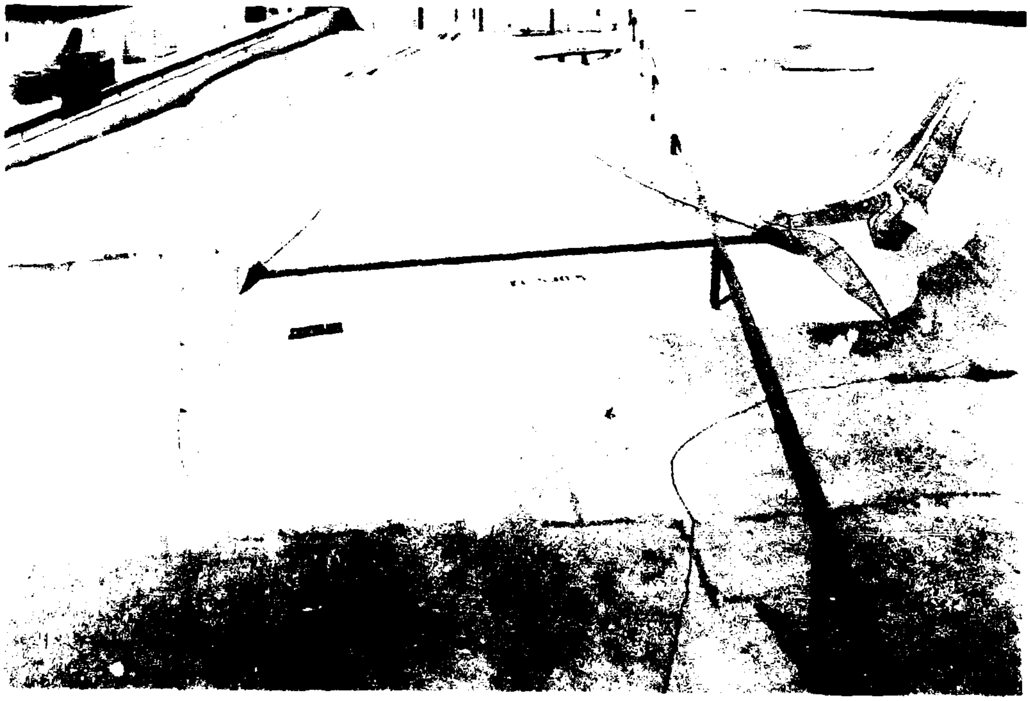
Scale Relations

9. The accepted equations of hydraulic similitude, based on the Froude criteria, were used to express mathematical relations between the dimensions and hydraulic quantities of the model and prototype. General relations for the transference of model data to prototype equivalents are presented as follows:

Characteristic	Dimension*	Scale Relations
Length	L_R	1:80
Area	$A_R = L_R^2$	1:6,400
Velocity	$V_R = L_R^{1/2}$	1:8.944
Time	$T_R = L_R^{1/2}$	1:8.944
Discharge	$Q_R = L_R^{5/2}$	1:57,243

* Dimensions are in terms of length.

a. Looking upstream



b. Looking downstream

Figure 2. General view of 1:80-scale model

Model measurements of discharge, water-surface elevations, and velocities can be transferred quantitatively to prototype by the scale relations.

PART III: TESTS AND RESULTS

10. Tests were conducted to determine approach conditions to the spillway, the hydraulic performance of the spillway, and general flow conditions in the exit channel for various discharges.

Approach Area and Spillway

11. Flow conditions observed in the approach to the spillway were found to be unsatisfactory with discharges Q of 200,000, 400,000, and 615,000 cfs (design discharge). The left and right 1V on 2H slopes upstream of the weir created unsymmetrical approach conditions to the spillway as shown in Photo 3. The location of the slopes in the approach forced flow around the slopes (Photos 4 and 5), which resulted in poor flow conditions approaching the spillway and reduced the effective length of the weir.

12. Approach conditions, along with the converging spillway design, caused large standing waves to form on the spillway. These standing waves exceeded proposed spillway wall heights with the design discharge of 615,000 cfs. Water-surface profiles along the spillway (Plates 3-5) along with photographs of flow conditions in this vicinity (Photo 6) indicated the nonuniformity of flow distribution on the spillway with the higher range of discharges. Water-surface profiles recorded perpendicular to the center line of the spillway with the design discharge (Plate 6) further indicate unsymmetrical flow conditions along the spillway. Velocities measured in the approach, along the spillway, and in the exit channel are shown in Plates 7-9.

13. Approach walls of different lengths (types 2 and 3 shown in Plate 10) were added to the existing walls in an effort to increase the efficiency of the spillway and reduce the height of the standing waves on the spillway. Very long approach walls were installed to produce optimum streamline approach conditions to the spillway. A reduction in height of the standing waves was observed with the additional wall lengths, but little difference in the head on the spillway was noted (Plate 11). These data indicated the extended approach walls resulted in little improvement in capacity and flow conditions along the spillway.

14. The left and right abutments of the spillway were modified in an attempt to improve approach conditions at the spillway. These modifications

included the addition of dikes extended upstream from the abutments and parallel to the center line of the spillway, which was designated the type 4 design spillway approach. Details of the configuration and location of dikes with respect to the spillway are shown in Photos 7 and 8 and Plate 12.

15. With the type 4 design spillway approach installed, flow conditions were satisfactory at the left and right abutments to the spillway. With the design discharge, the majority of flow around the left dike (looking downstream) was directed along the left spillway wall (Photo 7b). Flow conditions around the right dike were considerably different, as compared with the left dike, due to the direction of flow approaching the spillway. A significant amount of flow approaches the spillway from the right side due to the alignment of the spillway with respect to the reservoir. With the original design, the existing topography caused the flow to make a sharp turn around the right abutment just upstream of the spillway. Even though this same condition existed with the right dike in place (Photo 8), a definite improvement in approach conditions was noted at the right abutment and along the right side of the spillway. Water-surface profiles along the spillway (Plate 13) also indicated an improvement in flow conditions along the left and right spillway walls. A slight reduction in head on the spillway was noted with this design as compared to the type 1 design spillway approach (Plate 11).

16. It was observed that as the flow increased on the spillway, the water surface exceeded proposed wall heights and a portion of the flow rode along on top of the berm and entered the exit channel on the outside of the walls in the vicinity of the flip bucket. This could cause possible scour to develop along the wall's footings, resulting in potential failure of the walls.

17. Tests were conducted with walls of different lengths placed on top of the 6-ft-wide berm at the downstream end of the spillway chute in an effort to redirect flow back onto the spillway when the water surface exceeds the existing spillway walls. Wall lengths ranging from 40 to 120 ft were tested with a discharge of 615,000 cfs. A 120-ft-long wall having a warped surface (type 7 design berm wall) provided the best results in redirecting the majority of flow back onto the spillway. The wall heights along both sides of the spillway were also increased by paving the berm and existing topography. Flow conditions with the 120-ft-long wall modification and increased wall

heights are shown in Photo 9. Water-surface profiles in this area are provided in Plate 14.

18. No additional tests were conducted to further improve flow conditions on the spillway. The standing waves that were present for the various discharges observed were caused by the convergence of the spillway walls. Flow conditions could be improved, but this would entail major modifications to the spillway itself. Because this was not in the scope of testing in this investigation, efforts were directed towards documentation of flow conditions in the exit channel.

Exit Channel

19. With the original design, flow conditions observed in the exit channel were unsatisfactory, with significant standing waves present with the design discharge (Photo 10). A large eddy was observed between the spillway exit channel and the earth dam (Photo 11) with velocities ranging up to 11 fps along the toe of the dam with a discharge of 400,000 cfs (Plate 8). The maximum velocities recorded along the dam were caused by the large eddy and wave action (Photo 12) present in that vicinity. Velocities measured in the exit channel are provided in Plates 7-9.

20. The large eddy that was observed between the exit channel and the earth dam created a significant differential on the right wall at the downstream end of the spillway. The slower velocity and direction of flow created by the eddy resulted in a buildup of flow on the back side of the wall with high velocities and a much lower water surface on the inside of the wall where the flip bucket is located, as shown in Photo 10c.

21. Tests were conducted to determine the effects on general flow patterns in the exit channel if the right wall had failed. The eddy, which was present on the right side of the exit channel with all discharges observed, concentrated more of the flow along the left side of the exit channel without the wall in place, resulting in even worse flow conditions in the exit channel than previously observed (compare Photos 10c and 13).

22. Little dissipation occurred downstream of the flip bucket for the discharges observed. No noticeable backwater effects were present due to the bridge constrictions and highways located in the exit channel. The flow

simply rode over and through these areas without having any significant impact such as backwater effects developing.

PART IV: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

23. Tests were conducted to determine if the proposed modifications to the existing dam and spillway would be adequate to provide the necessary protection from a major flood event. Test results indicated that with certain modifications, the spillway would perform satisfactorily for the interim solution.

24. Unsymmetrical approach conditions to the spillway along with existing sloping topography caused contraction of flow at the abutments, which reduced the effective length of the spillway weir. Very long approach walls installed upstream of the weir significantly improved flow conditions to the spillway; however, this modification had little effect on reducing the head on the spillway. The long approach walls resulted in little improvement in spillway capacity and flow conditions along the spillway.

25. Dikes extending upstream from the left and right abutments to the spillway did improve flow conditions at the spillway weir. This resulted in increasing the effective length of the weir to a degree, but had little effect on significantly increasing the capacity of the spillway.

26. Large standing waves were present on the spillway due to approach conditions and the converging spillway design. These standing waves exceeded spillway wall heights proposed in the interim solution design. With the installation of the dikes in the spillway approach, a reduction in wave heights was noted, but additional wall heights were required to contain design flows within the limits of the spillway.

27. When the water surface exceeded the existing wall heights, the flow rode along on top of the berm and entered the exit channel on the outside of the downstream training walls. This could result in local scour along the bottom of the walls, possibly causing the walls to fail. A 120-ft-long wall having a warped surface placed on top of the berm at the downstream end of the spillway chute provided the best results in redirecting the flow back onto the spillway. Additional wall height was obtained by paving a small berm and existing topography along each side of the spillway.

28. No additional tests to try to reduce the height of the standing waves present were conducted, because these waves were created by the

convergence of the spillway walls. Major design changes to the existing spillway would have to be undertaken to reduce the heights of the standing waves. Because this was not in the scope of testing in this investigation, efforts were directed towards documentation of flow conditions in the exit channel.

29. Large standing waves were present just downstream of the flip bucket, and a large eddy was observed between the spillway exit channel and the earth dam with the higher range of discharges. Maximum recorded velocities along the toe of the dam were approximately 11 fps and were observed with a discharge of 400,000 cfs. Due to the magnitude of velocities measured in this area, riprap protection should be considered along the toe of the dam.

30. With the higher discharges, little, if any, dissipation occurred downstream of the flip bucket. Due to the erodible material in the exit channel, which was not reproduced in the model, severe scour would probably occur resulting in possible failure of the bridges and significant damage to the highways just downstream of the structure. No noticeable backwater effects were observed from the constricted bridge crossings and highways just downstream of the spillway. The higher flow simply rode over, around, and through these areas.

31. Because of the eddy present between the exit channel and spillway, a large differential in the water surface was present between the inside and outside of the right wall at the downstream end of the spillway. This was caused by the buildup of flow on the outside of the wall due to the direction of flow from the eddy and the high velocities and low water surface present in the flip bucket on the inside of the wall. Because of this differential, tests were conducted to determine the effects on flow conditions downstream of the structure if the right wall failed. Test results indicated that the eddy, present on the right side of the spillway, forced the flow from the spillway along the left side of the exit channel. This resulted in even worse flow conditions present in the exit channel than previously observed.

Recommended Design

32. Based on test results the modifications required to provide adequate protection during a major flood event consisted of (a) placing dikes immediately upstream of the left and right abutments of the spillway, (b) increasing

the heights of the spillway walls using existing topography, and (c) constructing walls on top of the 6-ft-wide berm at the downstream end of the spillway. These modifications, shown in Plate 15, should provide the protection needed for the interim solution. However, major changes such as increasing the spillway width would be required to improve flow conditions along the spillway and in the exit channel.

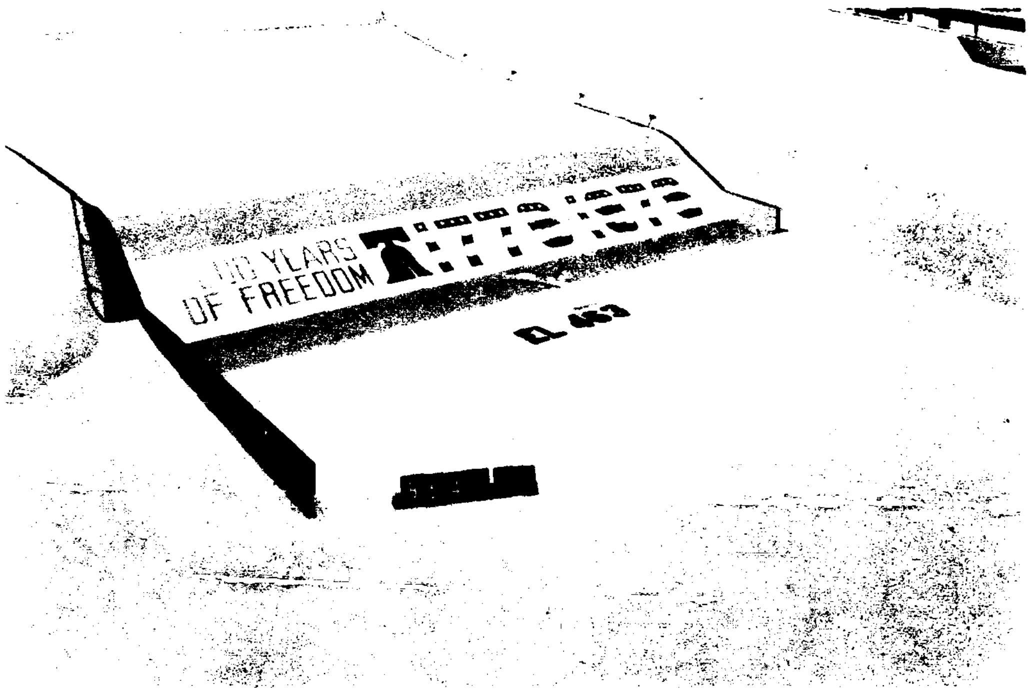


Photo 1. Spillway and flip bucket

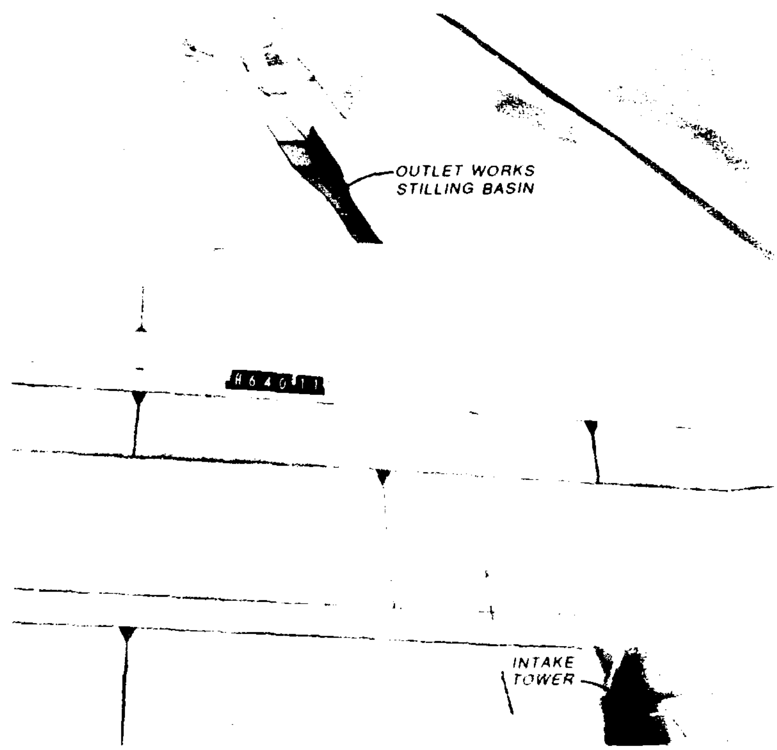
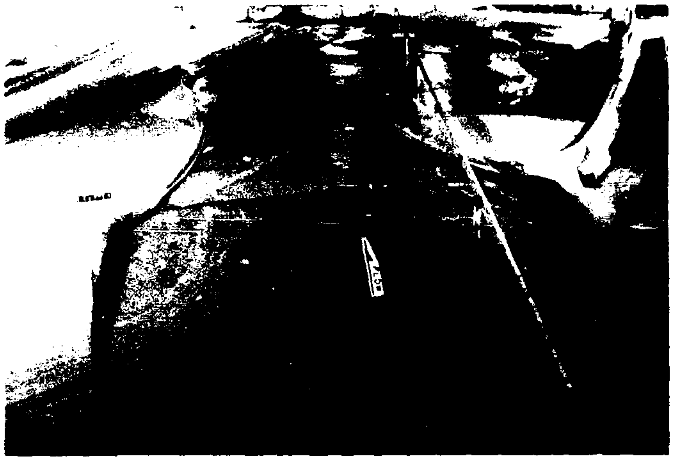


Photo 2. Intake tower and outlet works through dam

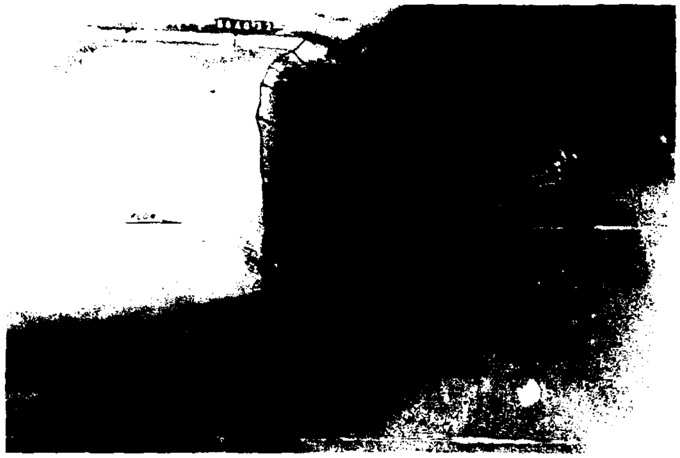


a. Q = 200,000 cfs

b. Q = 400,000 cfs

c. Q = 615,000 cfs

Photo 3. Flow conditions in spillway approach. Confetti illustrates surface flow patterns (exposure time 18 sec prototype)



a. Q = 200,000 cfs

b. Q = 400,000 cfs

c. Q = 615,000 cfs

Photo 4. Flow conditions (with dye) at left side of approach to spillway

a. Q = 200,000 cfs

b. Q = 400,000 cfs

c. Q = 615,000 cfs

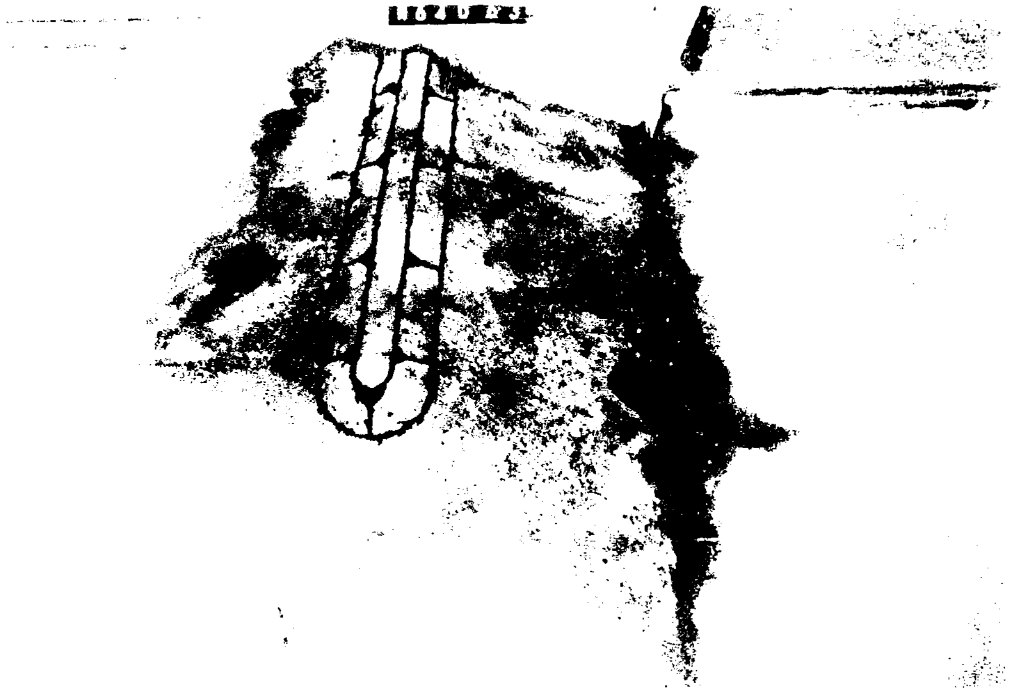
Photo 5. Flow conditions (with dye) at right side of approach to spillway

a. Q = 200,000 cfs

b. Q = 400,000 cfs

c. Q = 615,000

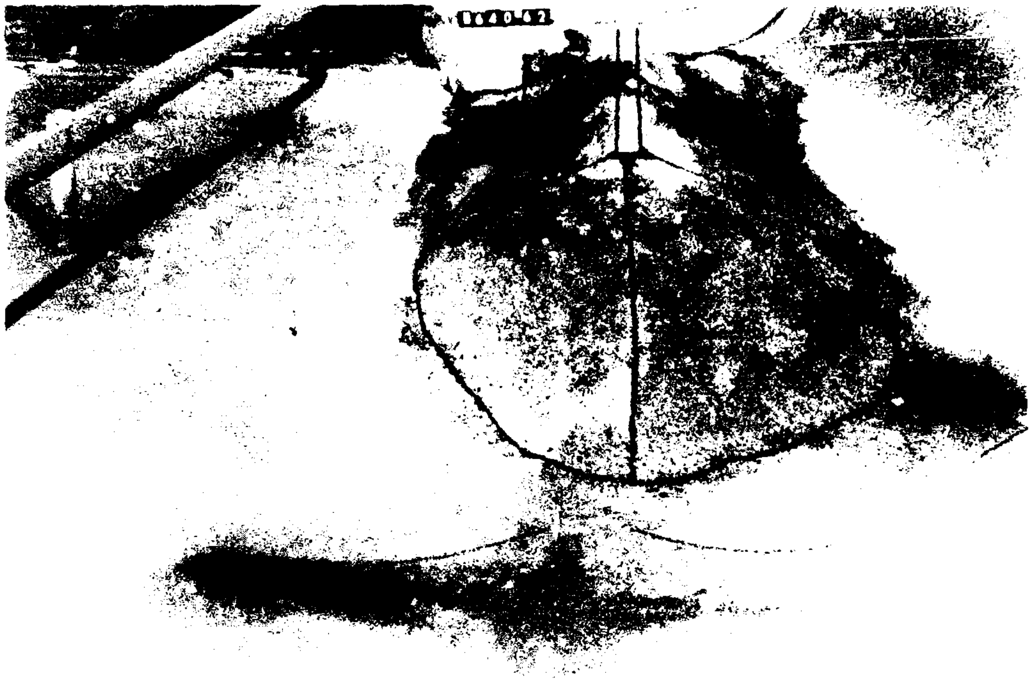
Photo 6. Flow conditions on spillway



a. Dry bed

b. Flow conditions around left dike. Confetti accents surface flow patterns. Q = 615,000 cfs (exposure time 9 sec prototype)

Photo 7. Left dike (type 4 design spillway approach)



a. Dry bed

b. Flow conditions around right dike. Confetti accents surface flow patterns. Q = 615,000 cfs (exposure time 9 sec prototype)

Photo 8. Right dike (type 4 design spillway approach)

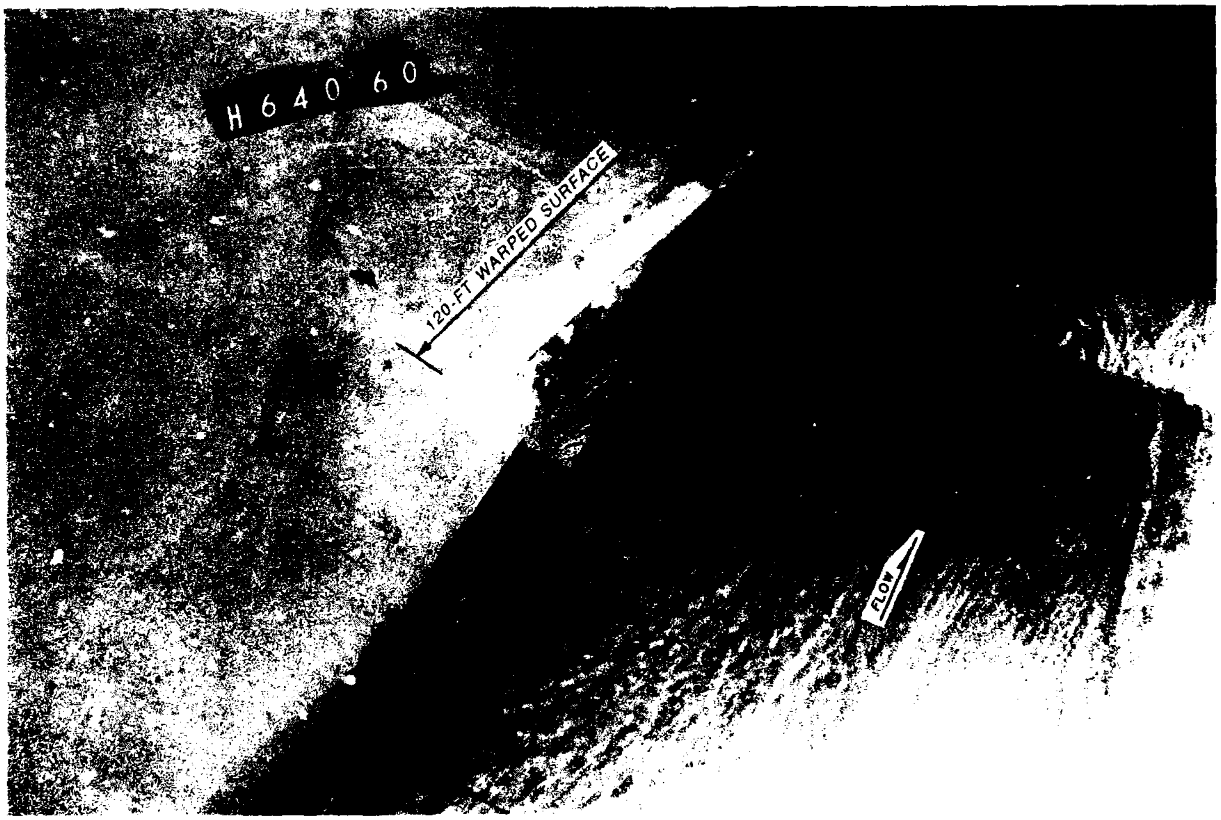


Photo 9. Warped-surface modification (between sta 19+00 and sta 20+20) recommended on top of existing wall. Q = 615,000 cfs

a. Q = 200,000 cfs

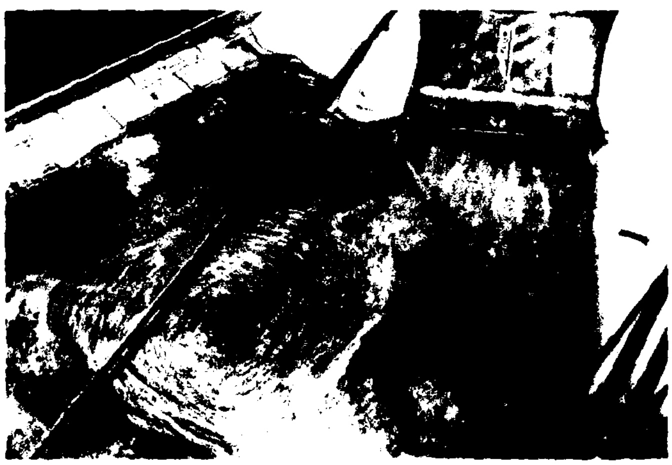
b. Q = 400,000 cfs

c. Q = 615,000 cfs

Photo 10. Flow conditions downstream of type 1 design spillway

a. Q = 200,000 cfs

b. Q = 400,000 cfs



c. Q = 615,000 cfs

Photo 11. General view of flow conditions in the exit channel. Confetti accents surface flow patterns (exposure time 18 sec prototype)

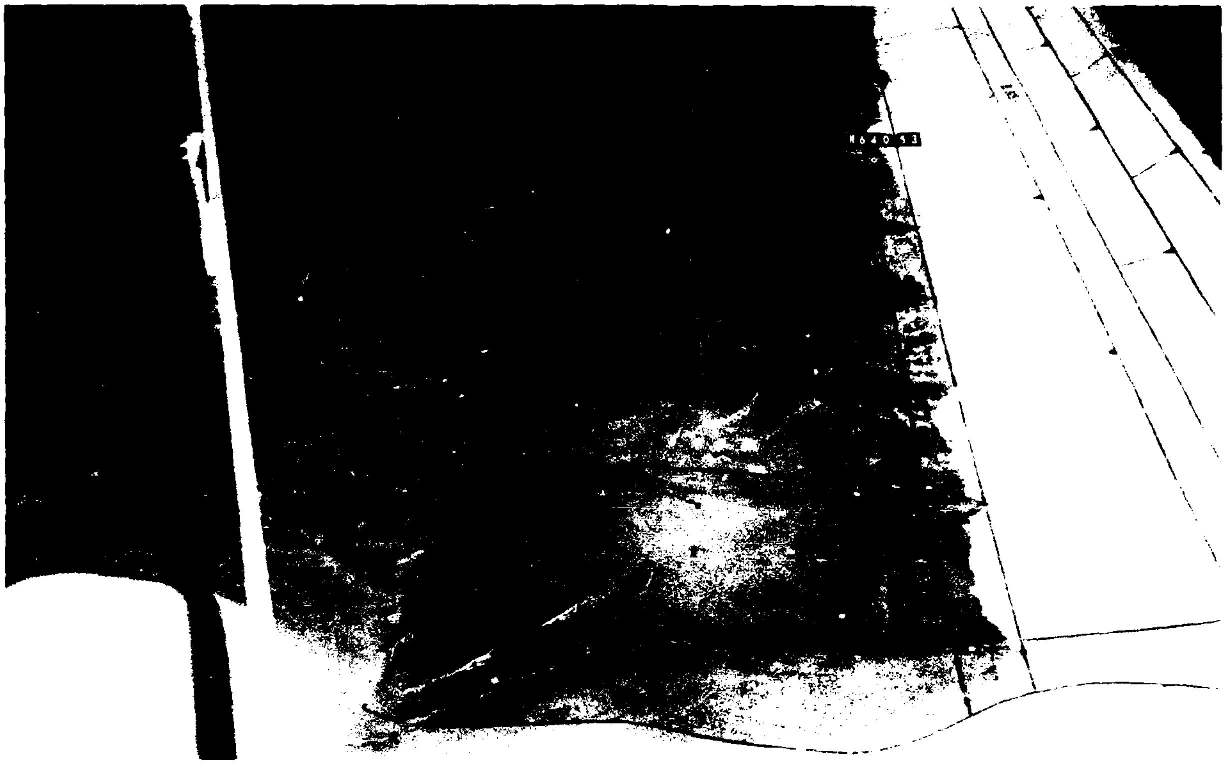
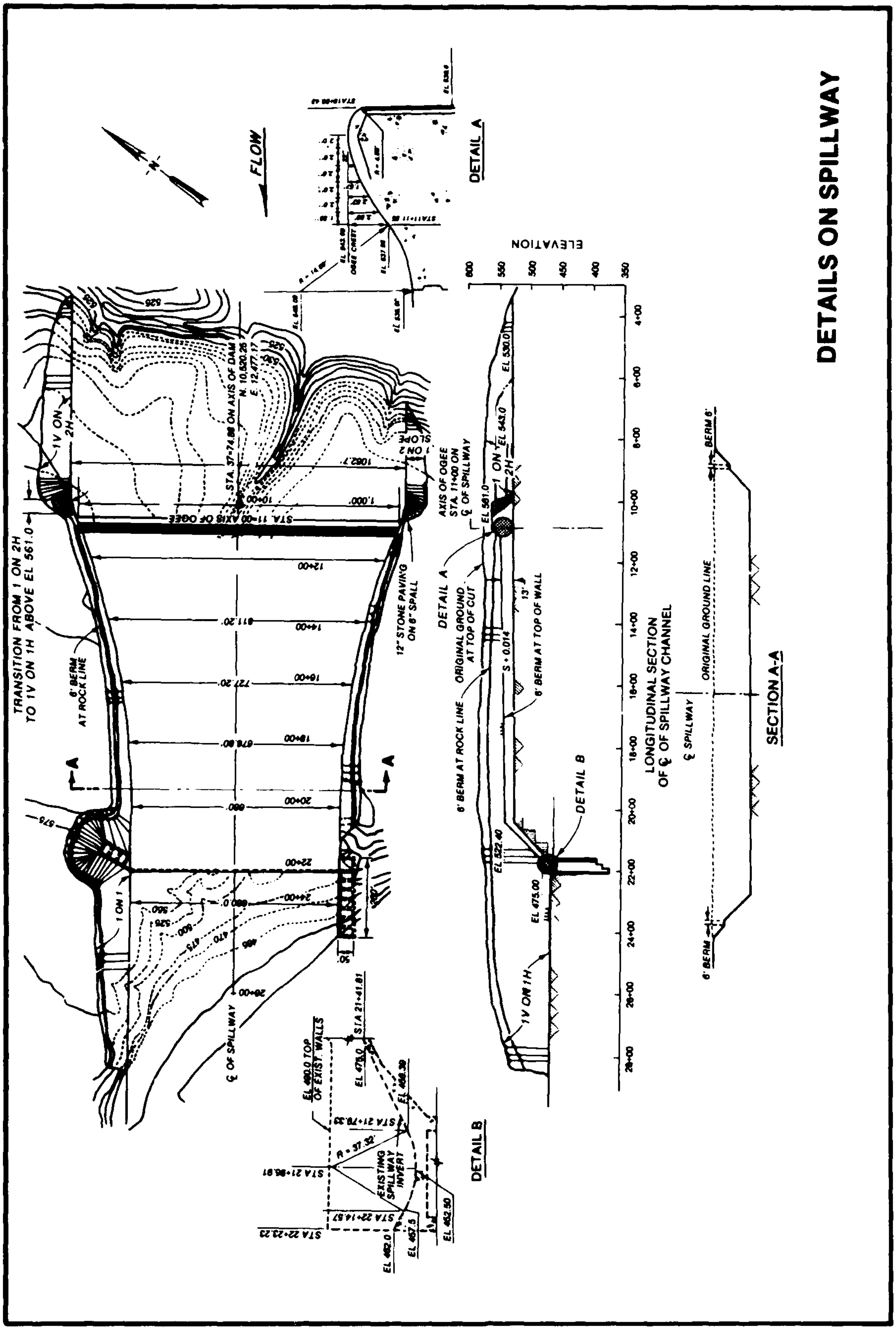
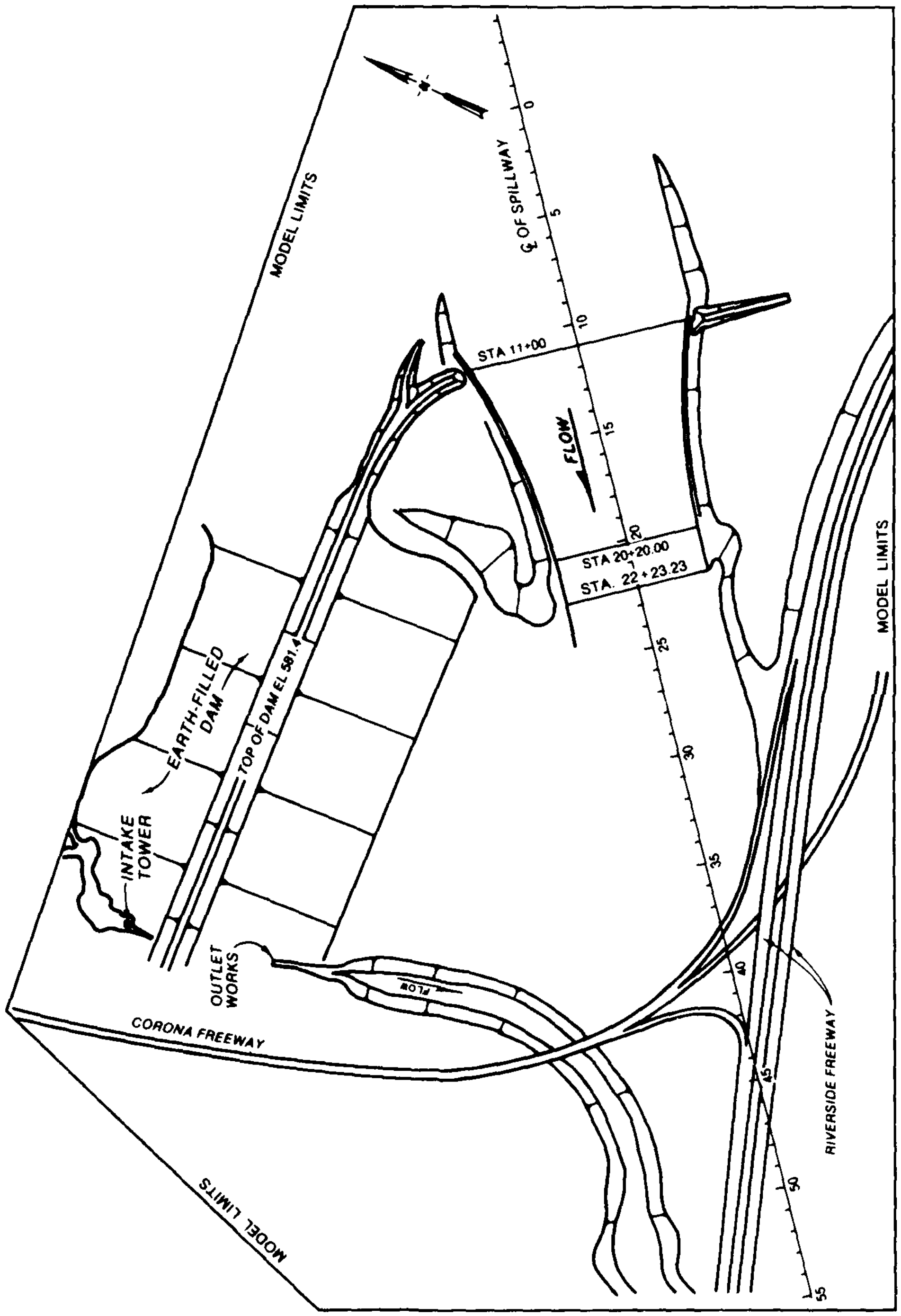


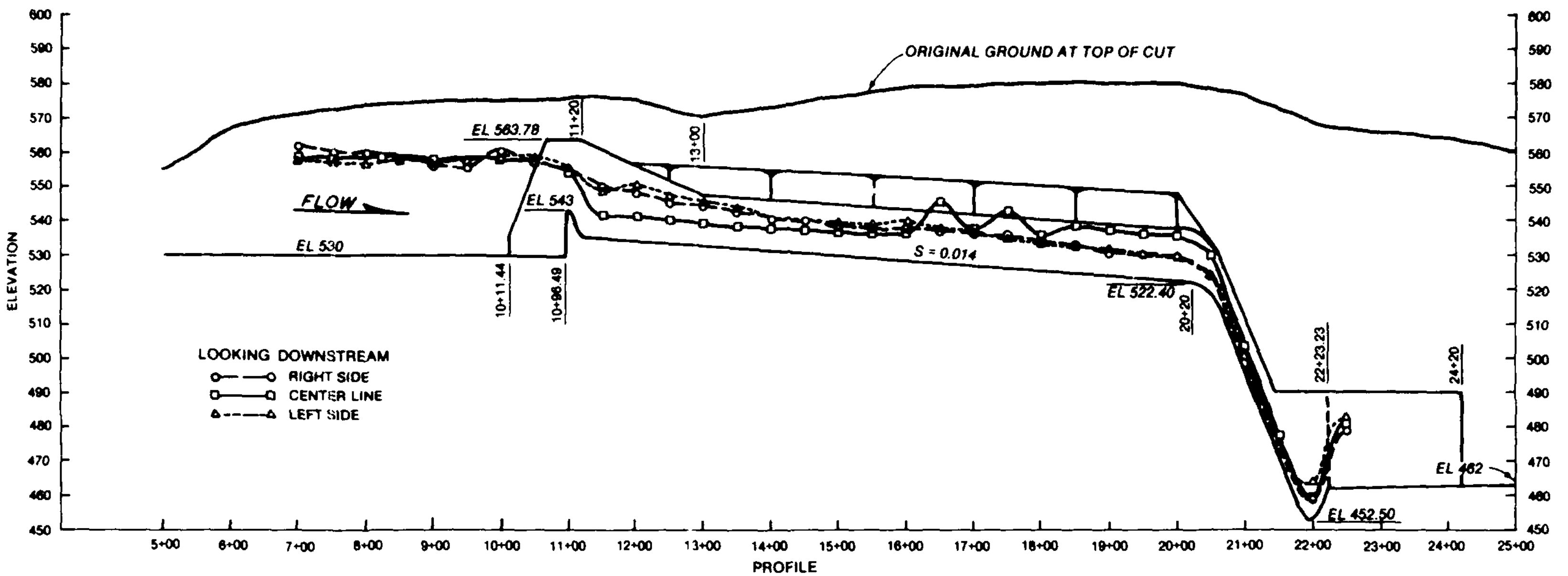
Photo 12. Flow conditions along toe of dam. Q = 400,000 cfs

Photo 13. Flow condition with right wall removed downstream of flip bucket. Q = 615,000 cfs


DETAILS ON SPILLWAY
FLOW
DETAIL A
DETAIL B
SECTION A-A
LONGITUDINAL SECTION OF ℄ OF SPILLWAY CHANNEL
ELEVATION
TRANSITION FROM 1 ON 2H TO 1V ON 1H ABOVE EL 561.0
6' BERM AT ROCK LINE
12" STONE PAVING ON 6" SPALL
STA. 11+00 AXIS OF OGEE
STA. 37+74.88 ON AXIS OF DAM
N. 10,520.26
E. 12,477.17
AXIS OF OGEE STA. 11+00 ON ℄ OF SPILLWAY
ORIGINAL GROUND AT TOP OF CUT
6' BERM AT TOP OF WALL
6' BERM AT ROCK LINE
S = 0.014
1V ON 1H
1 ON 2H
1 ON 1
1V ON 2H
SLOPE 1 ON 2
EL 561.0
EL 543.0
EL 530.0
EL 522.40
EL 475.00
℄ OF SPILLWAY
℄ SPILLWAY
ORIGINAL GROUND LINE
6' BERM
BERM 6'
EL 480.0 TOP OF EXIST. WALLS
STA 21+41.81
EXISTING SPILLWAY INVERT
STA 21+78.33
STA 21+96.91
STA 22+14.87
STA 22+23.23
EL 462.0
EL 467.5
EL 462.50
EL 458.30
R = 37.32'
1062.7'
1,000.'
811.20'
727.20'
678.80'
660.'
660.0'
10+00
12+00
14+00
16+00
18+00
20+00
22+00
24+00
26+00
28+00
4+00
6+00
8+00
350
400
450
500
550
600


MODEL LIMITS
℄ OF SPILLWAY
STA 11+00
FLOW
STA 20+20.00
STA. 22+23.23
EARTH-FILLED DAM
TOP OF DAM EL 581.4
INTAKE TOWER
OUTLET WORKS
FLOW
CORONA FREEWAY
MODEL LIMITS
RIVERSIDE FREEWAY
MODEL LIMITS
0
5
10
15
20
25
30
35
40
45
50
55

PLAN VIEW



WATER - SURFACE PROFILES
TYPE 1 (ORIGINAL) DESIGN
Q = 200,000 CFS

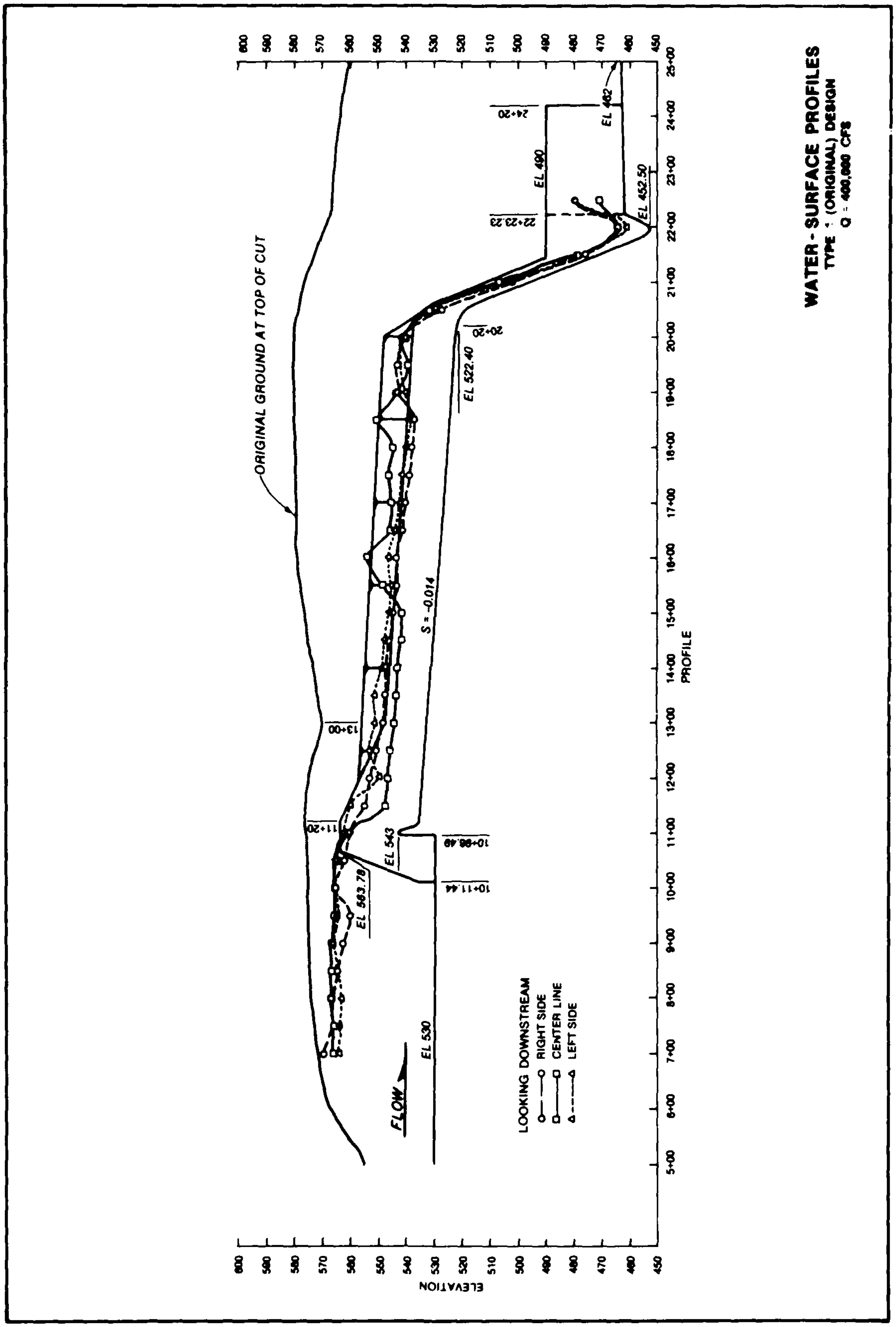
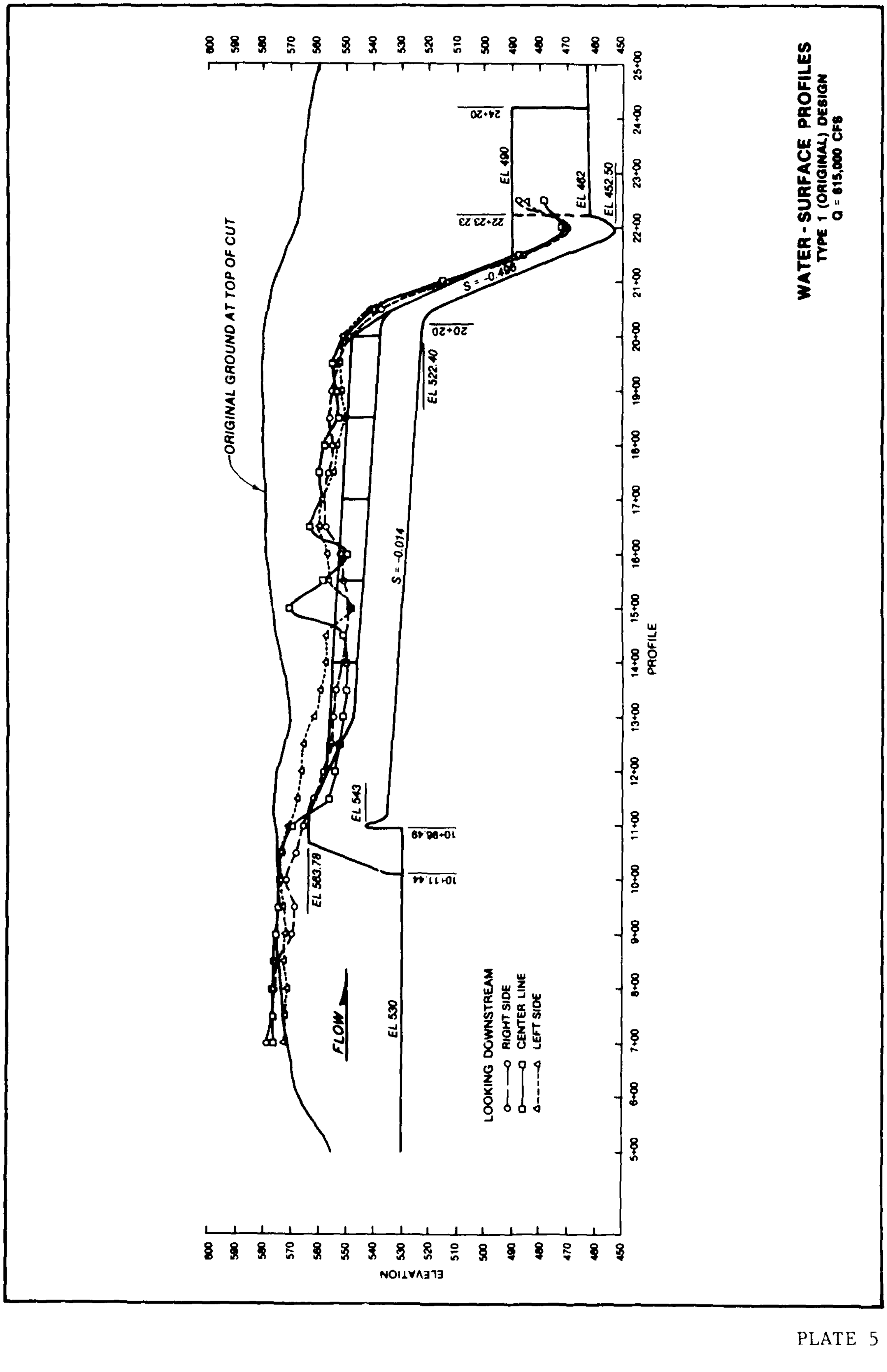
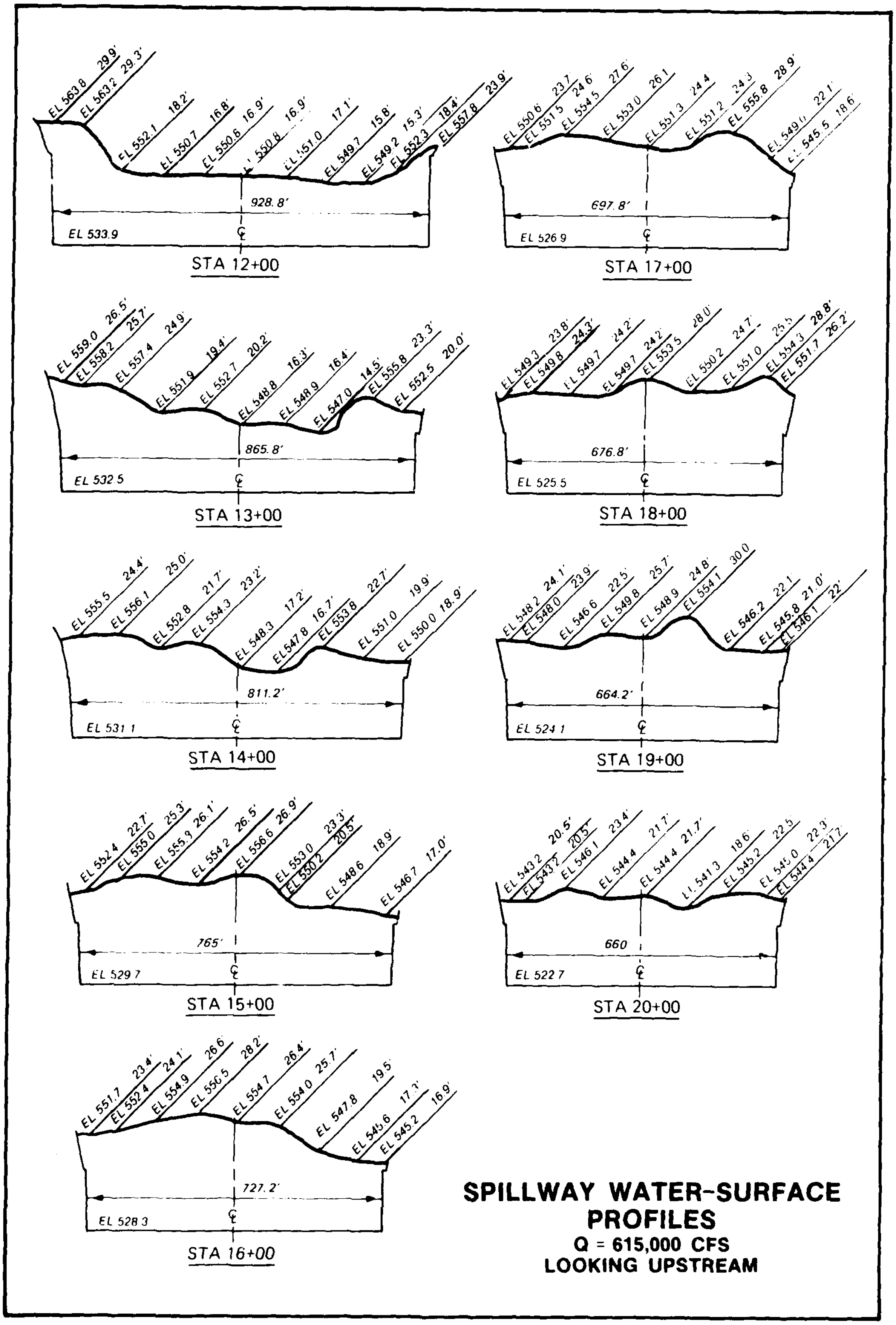

WATER - SURFACE PROFILES
TYPE 1 (ORIGINAL) DESIGN
Q = 400,000 CFS
ORIGINAL GROUND AT TOP OF CUT
LOOKING DOWNSTREAM
RIGHT SIDE
CENTER LINE
LEFT SIDE
FLOW
EL 530
EL 583.78
EL 543
10+11.44
10+98.48
11+20
13+00
S = -0.014
EL 522.40
20+20
22+23.23
EL 490
24+20
EL 452.50
EL 462
PROFILE
ELEVATION

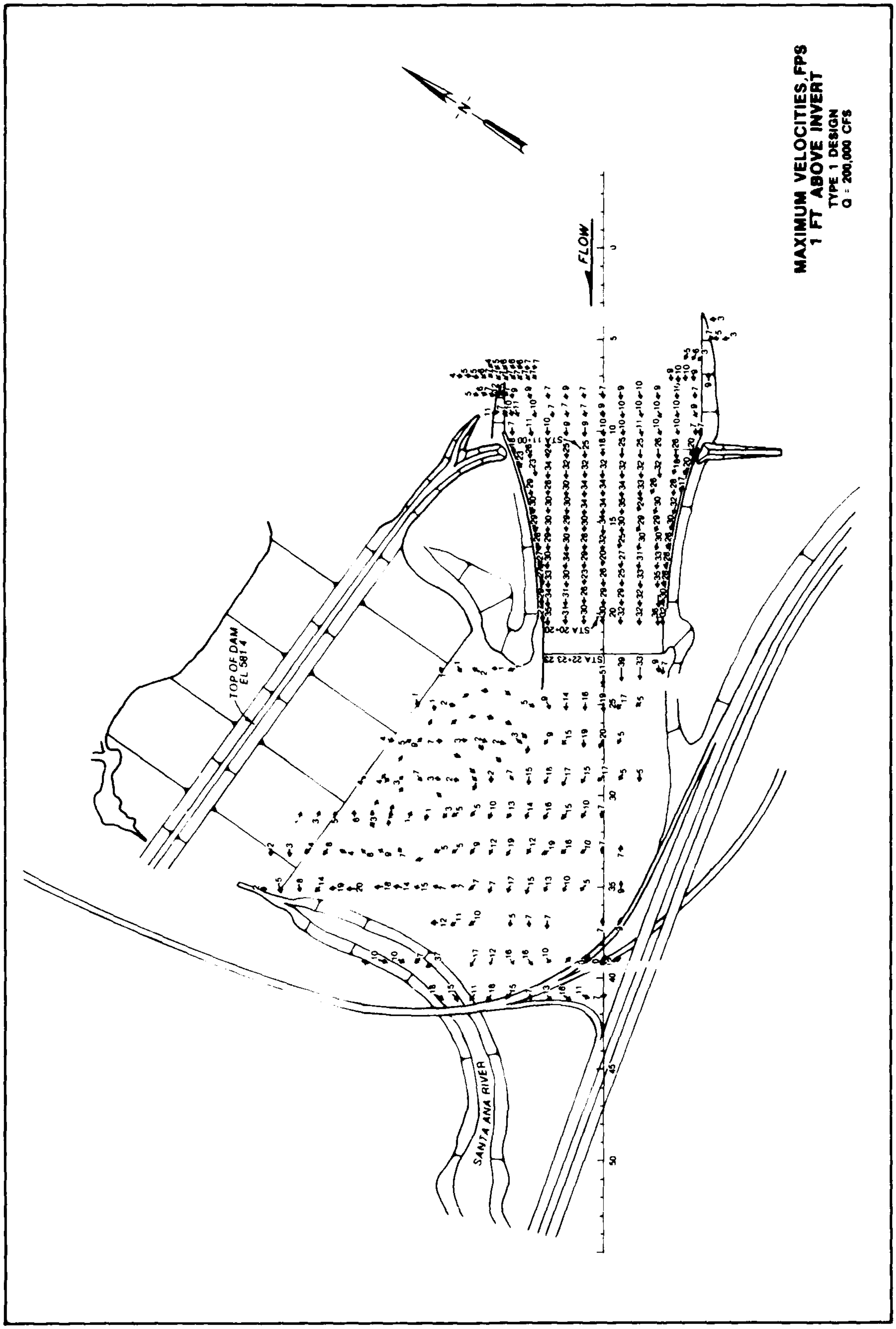
PLATE 4



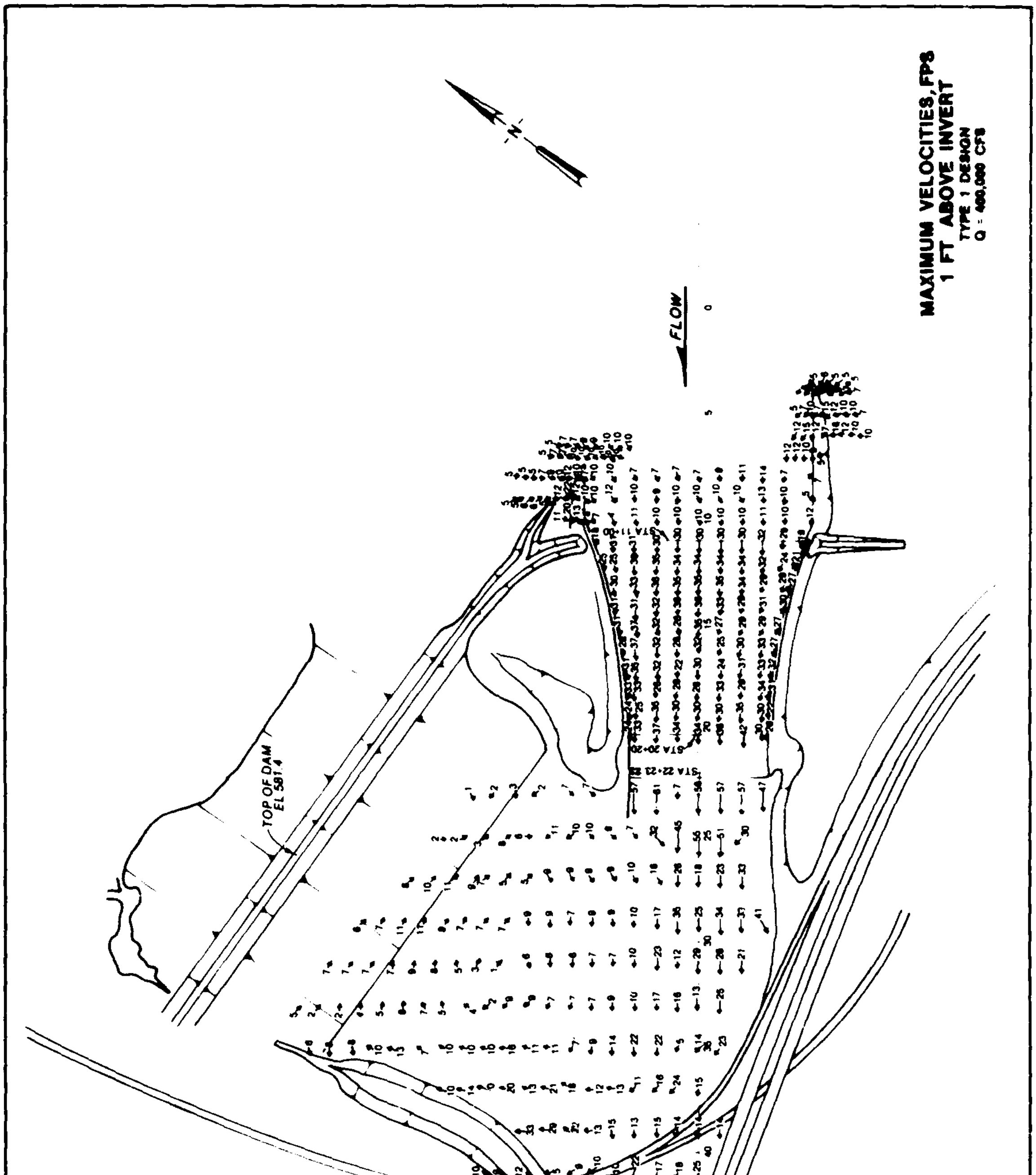
WATER - SURFACE PROFILES
TYPE 1 (ORIGINAL) DESIGN
Q = 615,000 CFS

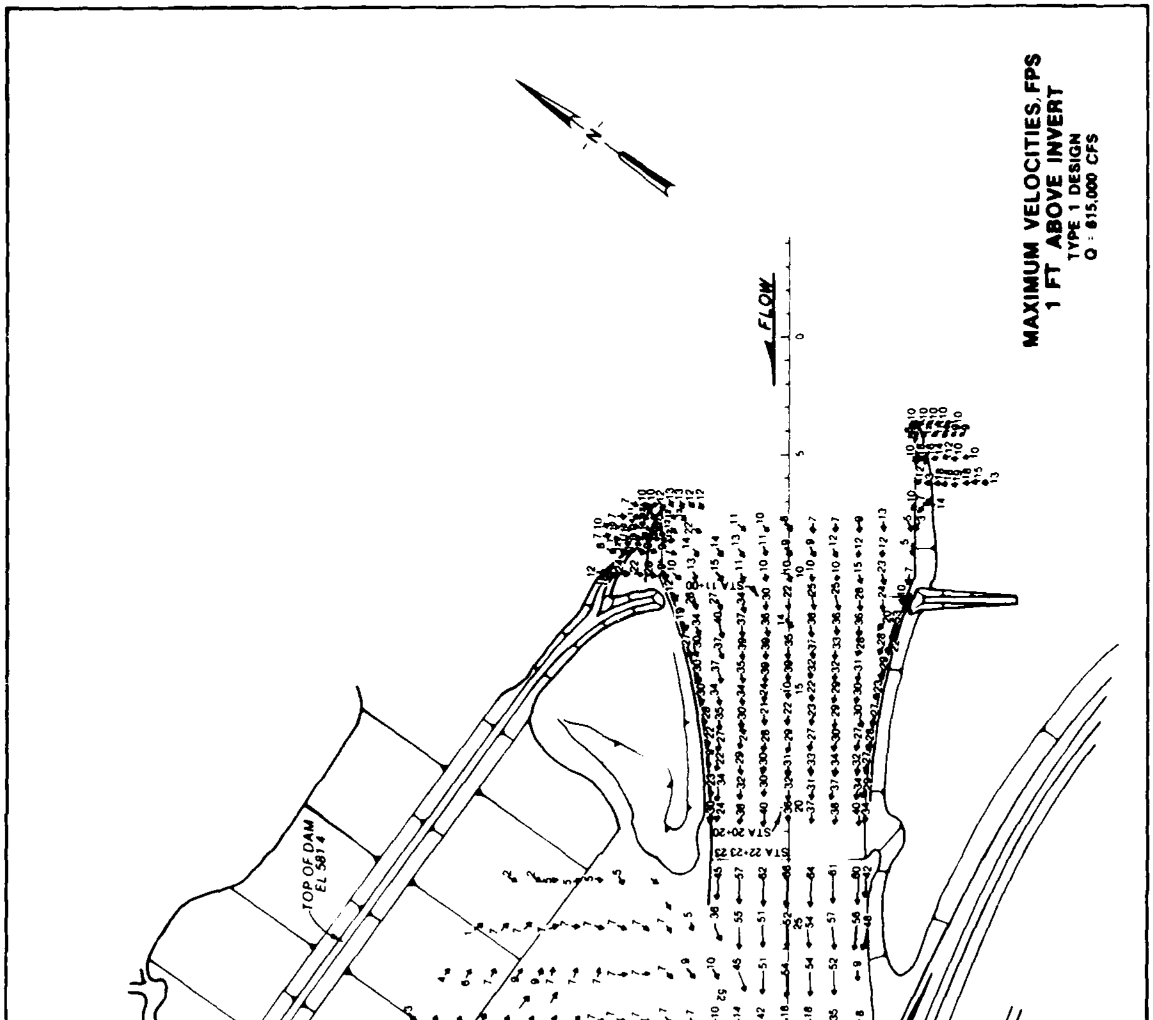


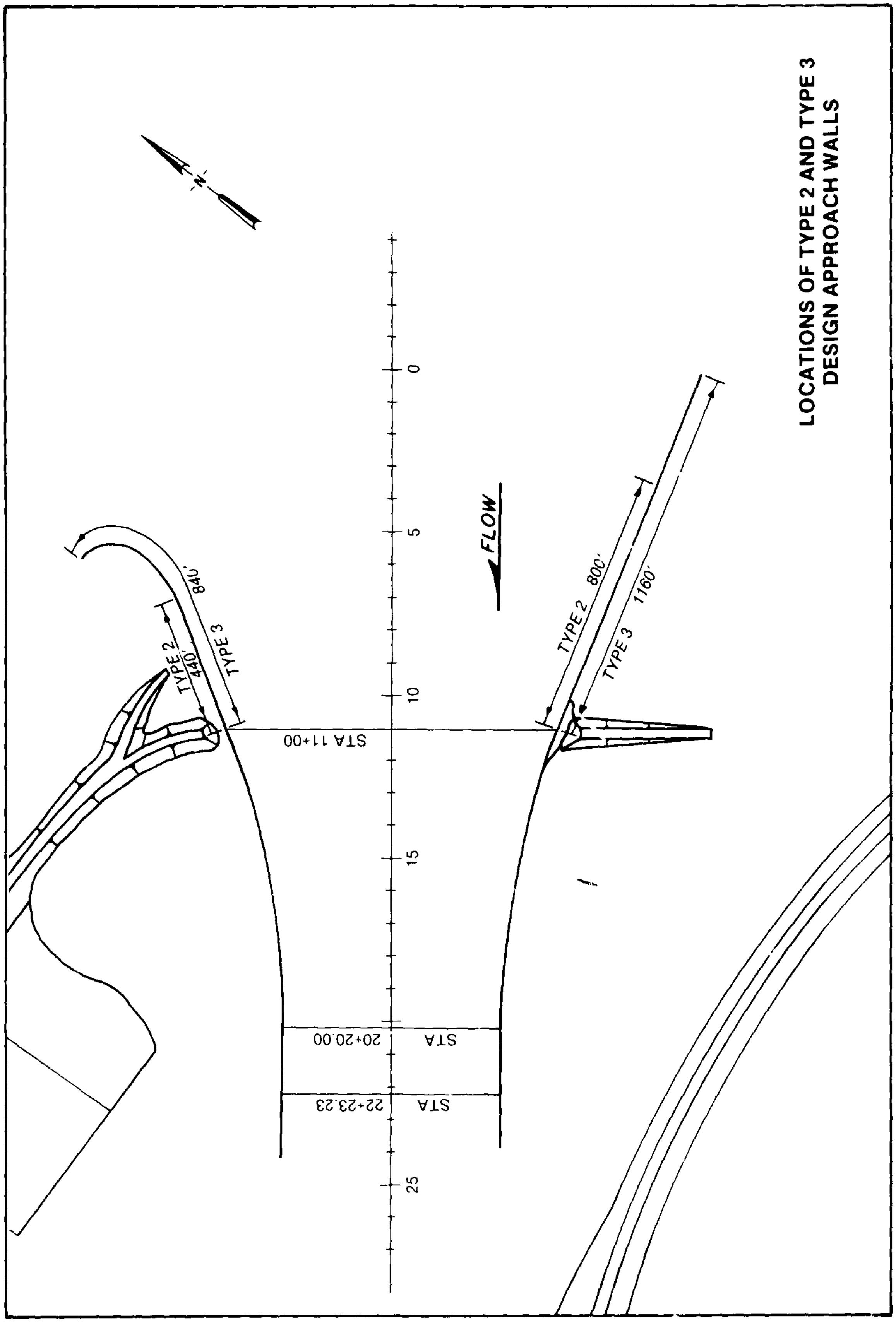
SPILLWAY WATER-SURFACE PROFILES
Q = 615,000 CFS
LOOKING UPSTREAM



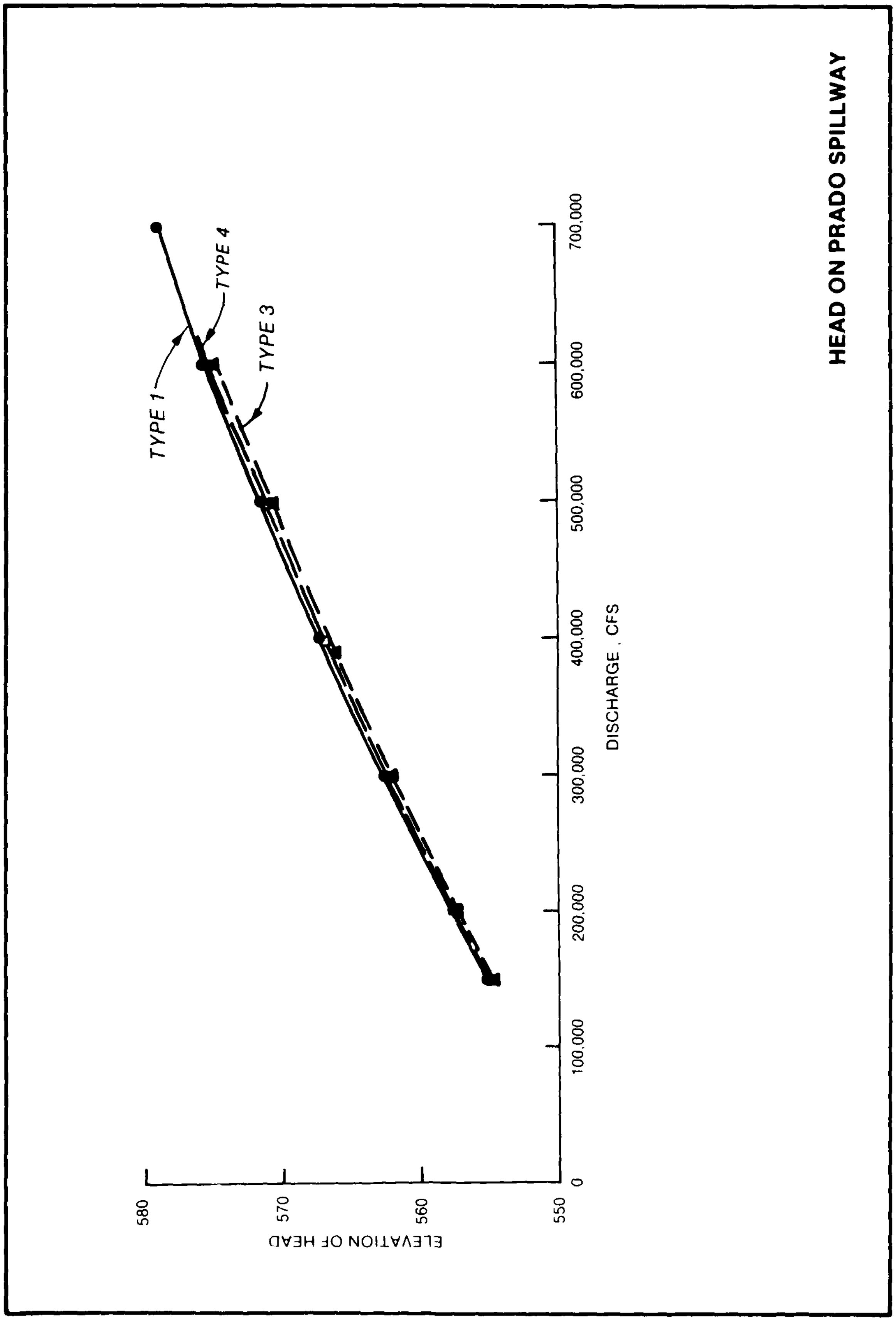
MAXIMUM VELOCITIES, FPS
1 FT ABOVE INVERT
TYPE 1 DESIGN
Q = 200,000 CFS


MAXIMUM VELOCITIES, FPS
1 FT ABOVE INVERT
TYPE 1 DESIGN
FLOW
N
TOP OF DAM
EL 581.4
SANTA ANA RIVER

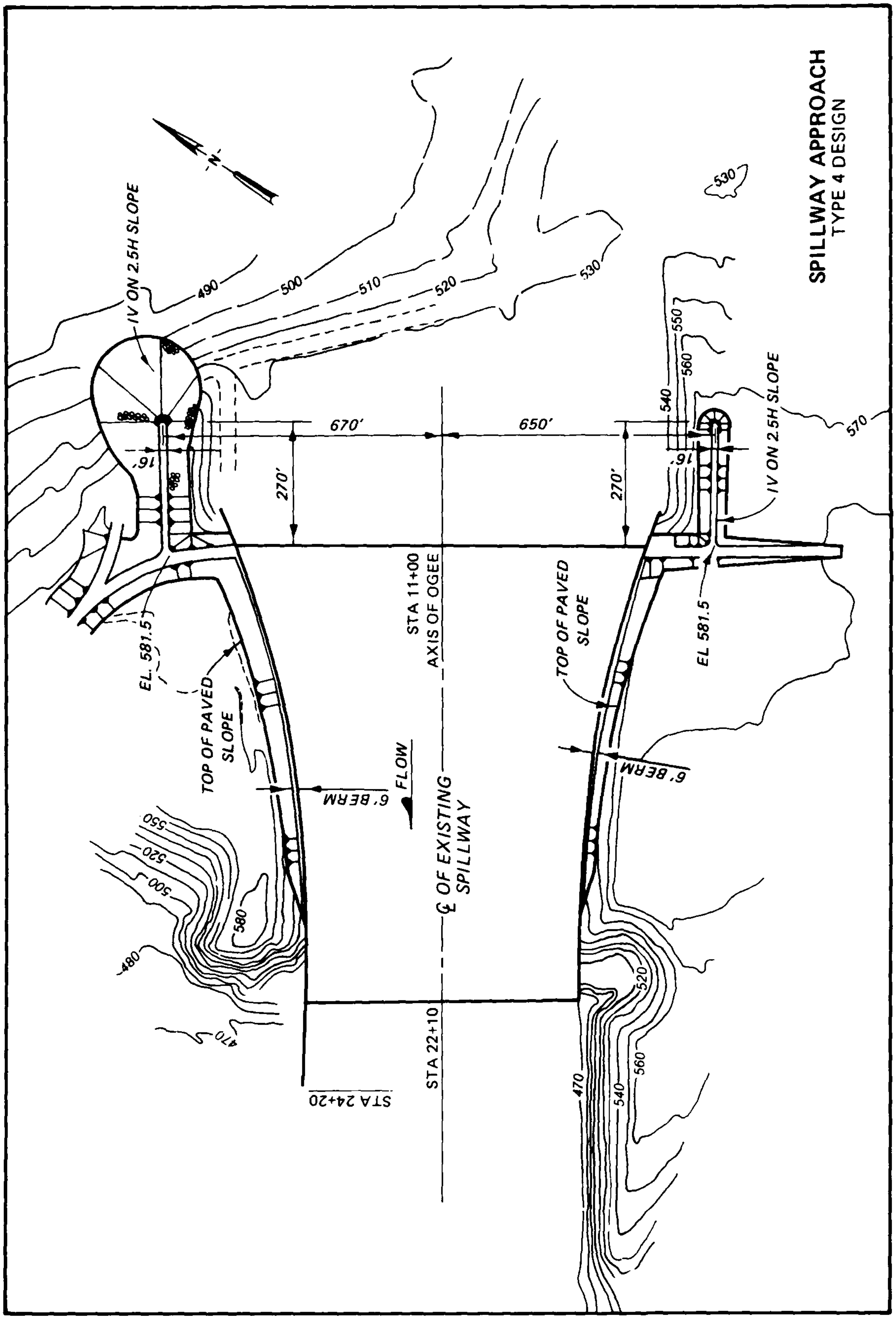

MAXIMUM VELOCITIES, FPS
1 FT ABOVE INVERT
TYPE 1 DESIGN
Q = 615,000 CFS
FLOW
N
TOP OF DAM
EL 581.4
SANTA ANA RIVER
STA 20+20
STA 22+23

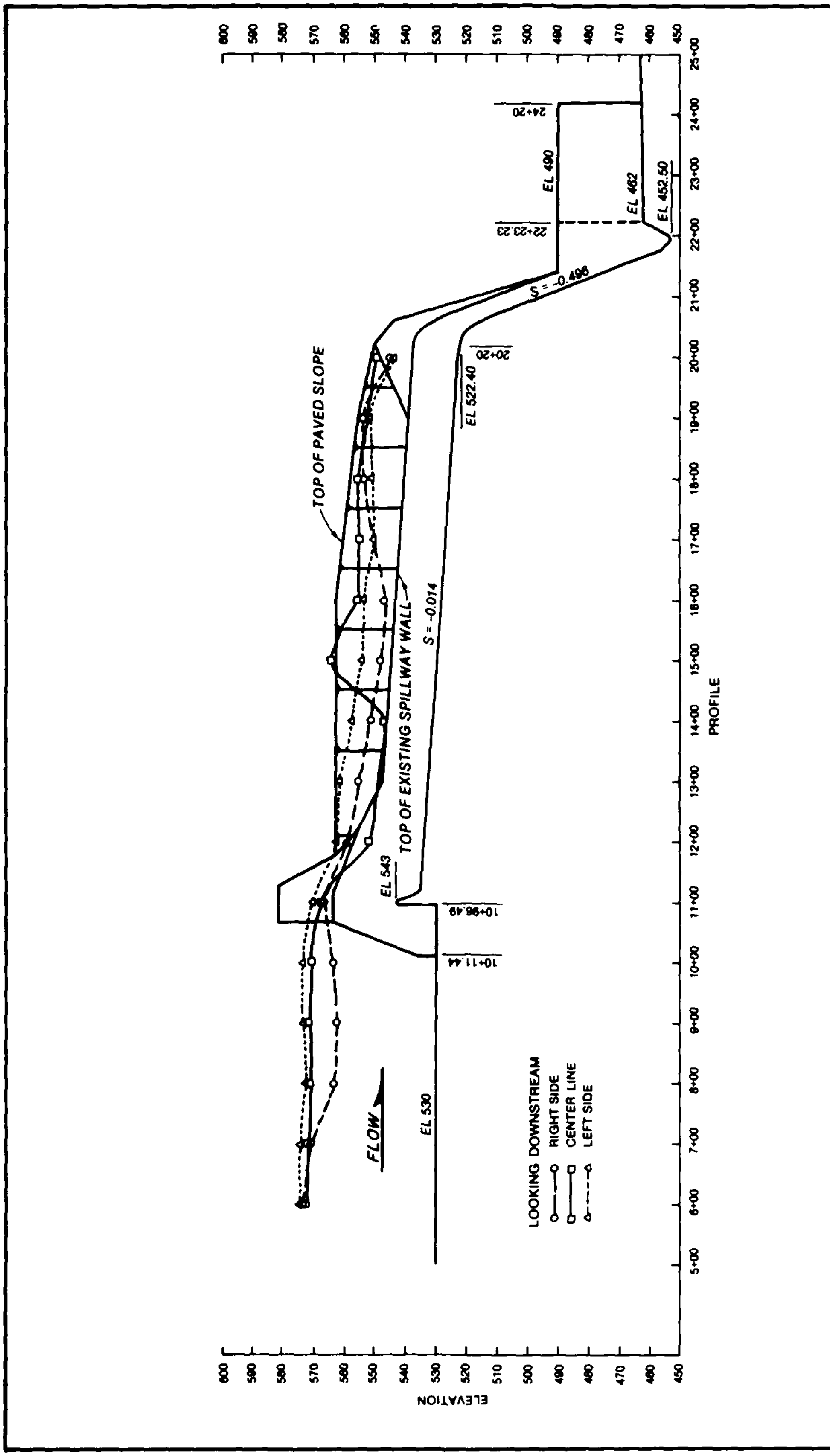


LOCATIONS OF TYPE 2 AND TYPE 3
DESIGN APPROACH WALLS

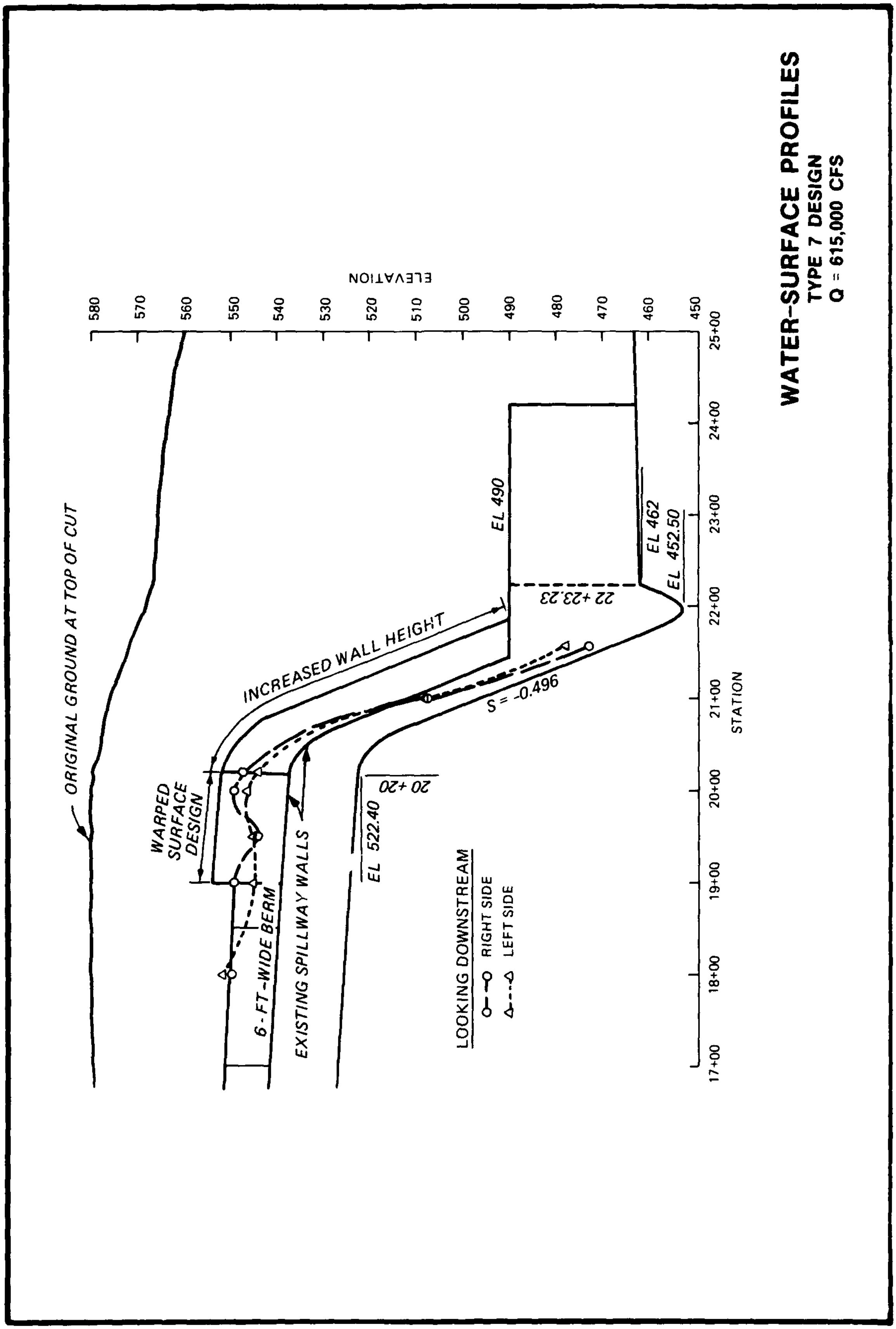


HEAD ON PRADO SPILLWAY

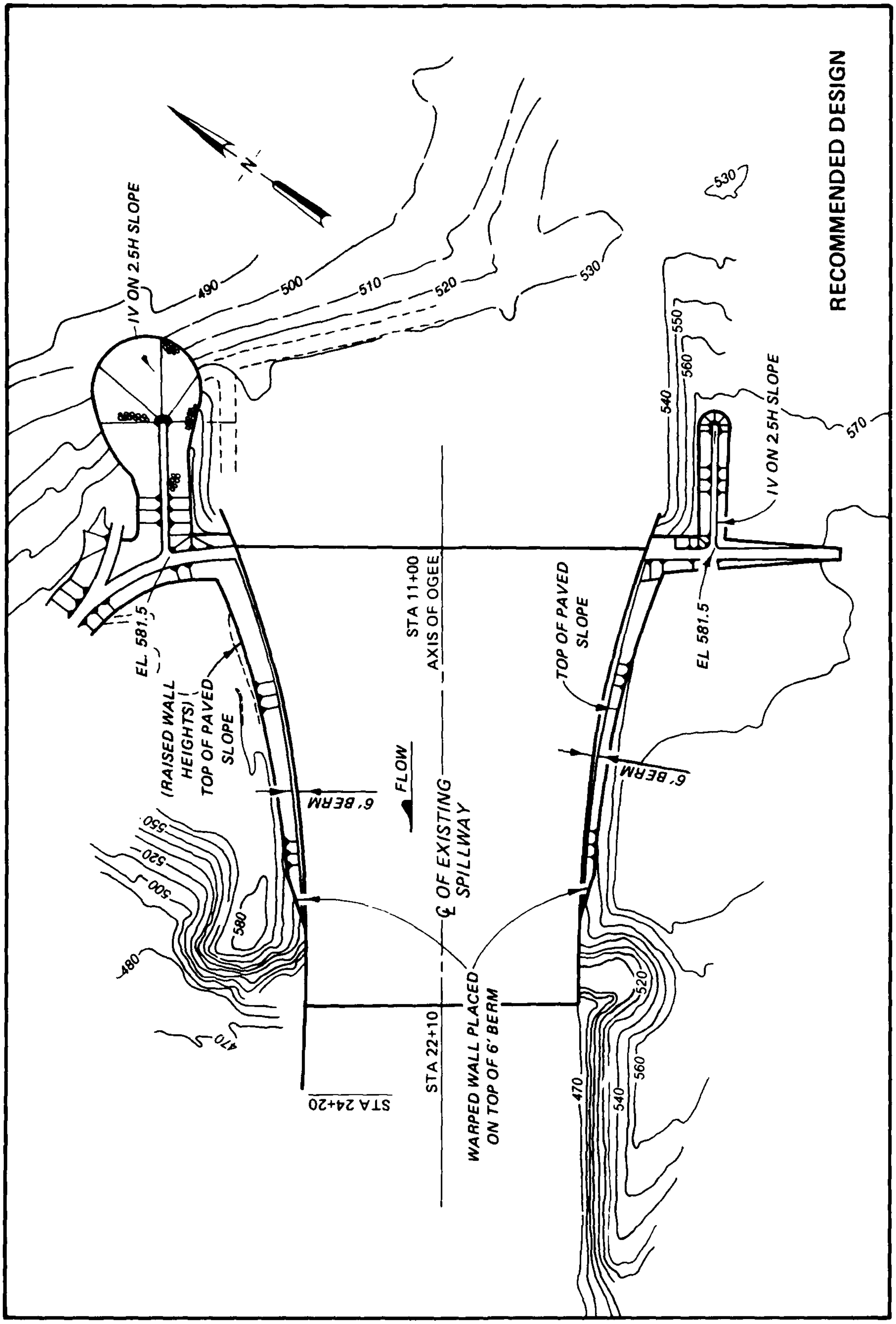

SPILLWAY APPROACH
TYPE 4 DESIGN
N
IV ON 2.5H SLOPE
IV ON 2.5H SLOPE
670'
650'
270'
270'
16'
16'
EL. 581.5
EL 581.5
TOP OF PAVED SLOPE
TOP OF PAVED SLOPE
6' BERM
6' BERM
FLOW
STA 11+00
AXIS OF OGEE
℄ OF EXISTING SPILLWAY
STA 22+10
STA 24+20
470
480
490
500
510
520
530
540
550
560
570
580



WATER-SURFACE PROFILES WITH THE TYPE 4 DESIGN SPILLWAY APPROACH
Q = 615,000 CFS



WATER-SURFACE PROFILES
TYPE 7 DESIGN
Q = 615,000 CFS


RECOMMENDED DESIGN
N
IV ON 2.5H SLOPE
490
500
510
520
530
530
540
550
560
570
IV ON 2.5H SLOPE
EL. 581.5
(RAISED WALL HEIGHTS) TOP OF PAVED SLOPE
STA 11+00
AXIS OF OGEE
TOP OF PAVED SLOPE
EL 581.5
6' BERM
FLOW
6' BERM
℄ OF EXISTING SPILLWAY
550
520
500
580
480
470
WARPED WALL PLACED ON TOP OF 6' BERM
STA 22+10
STA 24+20
520
470
540
560